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WAVE ENERGY STUDY

NEL OSCILLATING WATER COLUMN

BOTTOM STANDING DEVICE

PRELIMINARY STUDY

for

Department of Energy

Report No. PR7: Y5 DEY 2

May 1979

NATIONAL ENGINEERING LABORATORY

Department of Industry

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JUNE 1979

SUMMARY

The feasibility of a bottom standing oscillating water column wave energy device is studied. Several methods of providing foundations using existing technology are outlined. One method is selected and examined in greater detail. Recommendations for extension of the study are given.

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Appendix Drawing No. 568/PGA 21/1/A -
Oscillating Water Column, Bottom Standing Device.

1. INTRODUCTION

During the development of the Second Interim Reference Design for the National Engineering Laboratory's oscillating water column wave energy device (Reference 1) it became apparent that there would be considerable problems associated with the moorings and power transmission riser.

It was seen that these areas represent approximately 30% of the capital cost and a very high proportion of the continuing maintenance cost of the floating device. These costs could be avoided by a bottom standing structure.

Accordingly, this feasibility study was initiated by NEL to establish the basic concept for a bottom standing oscillating water column. It was assumed that the device would be situated off the west coast of the Outer Hebrides.

On the basis of the study carried out for the floating units, two principle design criteria were identified.

1. As little construction work as possible should be performed at the offshore site location.

2. The amount of structure required to resist long term hydrostatic pressures should be minimised.

2. GENERAL DESCRIPTION

The NEL Oscillating Water Column device extracts energy from the waves as follows:-

1. Primary System

   A column of water is induced to oscillate by the action of the waves. Its motion is used to pump air through the secondary system.

2. Secondary Conversion System

   (a) The oscillating air flow is rectified by louvre type valves to unidirectional flow.

   (b) This air flow drives a reaction turbine which in turn drives an electric alternator.

   (c) The alternating current produced is collected and transmitted to shore.

For the purposes of studying a bottom standing device, four water columns with associated equipment have been chosen to form one unit, with several units electrically linked to form a power station. Each unit is constructed in concrete with overall dimensions of 80m long, 30m wide and 27.5m high.

Construction of the concrete structure is carried out partly in a dry basin and partly at a sheltered inshore construction berth. Mechanical and electrical equipment is installed prior to the unit being towed out to its operating site.

At the offshore operating site, the device is ballasted down on to prepared foundations and secured against the 50 year storm by the use of rock anchors.

3. SITE CONDITIONS

A preliminary examination of possible sites off the Outer Hebrides was carried out by Rendel Palmer and Tritton (Reference 2). The most suitable sites at the 20m. water depth occurred at a distance of 2 - 3 Km. offshore.

Rendel Palmer and Tritton stated that published geological maps indicated that the bed rock at the chosen site would probably be Lewisian gneiss, with the possibility of intrusions of granitic type rocks. They found that the surface of the sea bed was rocky and undulating, with loose boulders and pockets of sand and other sediments.

The most significant plant found at the site is Laminaria Hyperborea (Kelp). It consists of a very tough stalk, known as the stipe, which terminates in a leaf like growth called the frond. The stipe grows over a period of six to seven years to lengths of 2 m and diameters of 80 mm at the site depth. The fronds also grow to similar lengths to the stipes. They are cast annually and thereafter decompose rapidly to very fine suspended particulate matter. The stipes attach to solid horizontal or gently inclined substrates using powerful sucker attachments known as hold fasts. During its early years the plant generally grows new hold fasts annually, which ensures that it maintains its grip on the rock. However, as it gets older the hold fast growth is reduced and eventually the plant becomes detached. It then drifts along the seabed until it either decomposes or is cast ashore. It can take up to six months for the stipe to decompose completely.

3. SITE CONDITIONS (Cont'd)

The maximum size to which the kelp grows is severly limited by the amount of light available. Its population and size falls off rapidly after the 25m depth. Its growth is also inhibited by drifting sand, which prevents completion of the fertilization procedure.

There are a number of other plant species present at the site, but with the exception of the large sponge Halichondria Panicea, none are expected to cause significant fouling. The sponge, although it could reach lengths of up to 1½ m, is soft and is easily detachable.

Lewisian gneiss is a massive metamorphic rock of the Pre-Cambrian period formed mainly from feldspar and quartz, with a banded texture called foliation. This latter characteristic makes the gneiss susceptible to weathering, although this is not expected to be a problem at the Hebridean site once below the surface of the rock. Sound gneiss can allow very high foundation bearing pressures. Typical physical and mechanical properties are given in Table 1 (from Chap. 9 of Reference 3).

4. FOUNDATIONS

Prior to the installation of a multi-unit power station, the site would be surveyed and a foundation method selected to suit the local sea bed conditions.

Several possible foundation methods have been identified. These are as follows:-

1. A Levelled Sea Bed

   The sea bed is initially cleared of seaweed and loose material. A sloping or very uneven sea bed is levelled by dredging and blasting if necessary. Final levelling to tolerances which can be accepted by the structure is achieved by depositing a layer of crushed rock over the entire foundation area.

   The unit is then ballasted down into position and secured using rock anchors drilled into sound rock. A carefully selected rip rap blanket is placed round the periphery of the structure to prevent scour.

   This method has been used to provide foundations for Offshore Lighthouses in Sweden (Reference 6) and a Tidal Power Plant in the USSR (Reference 7).

2. Pile Foundations

   Hollow/...

2. Pile Foundations

Hollow circular piles are drilled a short distance into sound rock. The hole is extended below the toe of the pile and a concrete plug with integral ducts placed to secure the pile. The piles are then cut off at a designed level just above the sea bed. The unit, with its underside prepared to mate with the piles, is floated over and ballasted down on to the piles. Rock anchors are then placed through the ducts and stressed to secure the structure.

Similar methods have been used in foundations for bridges in Denmark and Japan (References 8 and 9).

3. Pier Foundations

This method is similar in principle to Method 2, but instead of a large number of piles a small number of piers are used to provide a level base on which to place the structure. Caissons, shaped to fit the sea bed contours, are set on the sea bed and filled to the designed level with concrete. The unit is ballasted on to the piers and any irregularities filled with grout. Rock anchors are then drilled through the pier to secure the structure.

Method 1 is preferred by the nominated Consultants because it is considered that the preparation of the sea bed and placing of a rock blanket would be less costly than the structural work required to achieve the necessary dimensional accuracy at the interface between the piles or piers and the structure.

4. FOUNDATIONS (Cont'd)

Accordingly, the rest of this report assumes that Method 1 has been adopted.

5. STRUCTURAL CONSIDERATIONS

A possible structural configuration is shown on Drawing No. 568/PGA21/1A. This arrangement, which is an evolution of the second interim reference design, reduces the amount of hydrostatic resisting structure by situating the on-board equipment entirely above mean sea level. A wide base is provided to reduce draft while the structure is floating during the construction stages, and to reduce the bearing pressure under the foundations after installation.

The size of the structure shown on the drawing is restricted to 80m long by 30m wide to minimise the problems associated with providing an acceptable foundation.

With the location of the mechanical equipment entirely above the sea level, there is very little permanent buoyancy in the structure. Sufficient buoyancy during construction and installation is provided by temporary bulkheads across the mouth of the water column and between the rear edges of the main transverse walls.

Although the arrangement of the equipment is more compact than in the floating reference design, it will suffer from increased air flow losses in the secondary conversion system. This is due to the increased number and complexity of the bends in the air ducts and also to the shorter length available for diffusing the exhaust from the turbine. There will also be increased mechanical losses due to the gear box necessary to provide a drive to the alternator and to the increased number of bearings in the system.

However, the compactness of the arrangement allows all the equipment and most of the ducting to be contained inside one chamber above the water column. The walls and floor of the chamber form part of the main structure, while the roof may be constructed using either hollow precast prestressed concrete beam sections (as shown on the drawing) or a structural steel lattice girder system.

The rock anchors required to secure the structure are situated in the base and surrounding walls of the water column chamber. The preliminary stability calculations show that 130 vertical and 56 inclined medium capacity anchors are required for the 80m long structure shown. Each anchor provides a force of 3610 kN using a 19/18 Dyform prestressing cable.

Current design trends in rock anchor technology are leading to anchor capacities in the order of 10 - 20 MN (see Reference 13). However, anchors of this size require high strength tendons which are very bulky and require considerable constructional space. Therefore medium capacity anchors were chosen for ease of handling in the restricted installation areas available, particularly within the water column chamber.

A removable hatch situated above one of the air duct openings in the roof of the chamber allows access for the rock anchor installation plant. The precast floor slabs provide a working platform and protection to the anchorage points.

Although/...

5. STRUCTURAL CONSIDERATIONS (Con't)

Although the positioning of anchors in this part of the structure increases the complexity of the installation procedure, it facilitates the use of inclined anchors andhelps to distribute the foundation pressures more evenly.

The larger number of medium capacity anchors also help to distribute the compressive prestress effect through the structure. The vertical anchors in the walls provide a direct compressive stress which reduces crack widths occurring under serviceability loadings. This allows the amount of ordinary reinforcement to be reduced.

Corrosion protection to the anchors is provided by a double system which consists of a cement grout surrounding either a high strength epoxy or polyester resin coating in the fixed length or a water resistant grease packed plastic sheath in the free length. The anchorages in the structure will be covered with filled caps and enclosed in epoxy resin blocks.

The cost and effectiveness of the rock anchors depends entirely on the soundness of the gneiss underlying the structure. At present there is no detailed information available about the geology of the selected site. A full geological site investigation will be required to enable further detailed design work to be carried out.

This study has concentrated on establishing the minimum overall structural dimensions for a water column with fundamental dimensions similar to those used in the second interim reference design. Further work is required to quantify how these dimensions are affected by the proximity of the sea bed and by the fixity of the structure.

6. CONSTRUCTION AND INSTALLATION

It is proposed that the units are built in a similar manner to concrete North Sea Oil production platforms. With the dimensions shown on Drawing No. 568/PGA21/1A it would be possible to construct several units at the same time in one basin, or to make use of one of the large shipbuilding or repair drydocks.

A construction sequence would be as follows:-

1. The base slab is first constructed in the dry basin.

2. The main vertical walls are then constructed using fixed shuttering and slipforming as appropriate to a height sufficient to allow float out from the basin.

3. Temporary bulkheads are then installed across the mouths of the water columns and along the rear edge. The basin is flooded and the structure is floated out.

4. At a sheltered inshore berth the walls are completed and the precast floor slabs are placed in position in the bottom of the water column chambers. The roof slabs to the chambers are then constructed.

5. Major mechanical plant items and the air ducting are installed and the roof of the machinery chamber is constructed. The mechanical and electrical installation work is then completed.

6. The unit is then ready to tow to the offshore operating site.

6. CONSTRUCTION AND INSTALLATION (Cont'd)

Preparation of the foundations at the Offshore site is carried out concurrently with the construction of the structure.

The preparation sequence is as follows:-

1. Seaweed, sand, boulders and other loose materials, and weathered rock is removed from the foundation area by dredging, scraping and high pressure water jets.

2. If necessary the area is levelled by drilling and blasting.

3. A layer of suitably graded crushed rock is placed to a minimum depth of ½ metre and levelled.

The installation of a unit is as follows:-

1. The structure is positioned over the prepared area.

2. The structure is ballasted down by flooding the buoyancy areas with water or heavy mud. The mud may be required to increase initial stability while the installation of rock anchors commences.

3. The vertical rock anchors in the walls are installed immediately after the structure is ballasted down.

4. While the vertical rock anchors are being installed, a cement grout is pumped into the crushed rock blanket. This prevents ingress of water during the installation of the inclined anchors in the base of the structure.

6. CONSTRUCTION AND INSTALLATION (Cont'd)

5. With completion of the installation of the vertical rock anchors one water column chamber at a time is emptied and the inclined rock anchors are installed.

6. During the installation period a rip rap blanket is laid around the periphery of the structure to prevent scour.

7. When all the rock anchors are installed and correctly stressed, the areas behind the temporary bulkheads are flooded and the bulkheads removed from the front and rear of the structure. The device is then commissioned.

For the particular configuration prepared for this report there is significant constraint to the installation procedure at stages 2 and 3. When the structure is initially ballasted down on to its foundation and prior to the installation of any rock anchors, the maximum height of wave that can be resisted is approximately 3m. With the provision of heavy mud inside the water column chambers this height is increased to 7m. Thus the early stages of installation will require a period of calm weather in order that they may be safely completed.

It may be possible to overcome this problem by incorporating extra chambers for solid ballast. If these were placed behind the water column they would increase the width of the base of the structure which would reduce the draft required for towout. However, the resulting wider base would also require a larger area of level foundation at the site.

7. MAINTENANCE

A bottom standing OWC device situated at the Outer Hebridean site may be subject to loss of efficiency caused by accumulation of sand and organic debris at the entrance, floor and rear bottom corner of the water column chamber, and by encrustation by barnacles, mussels and growth of sponges on the interior walls. However, it is understood that due to the lack of light and the presence of silt and sand the growth of the principal seaweed in the location i.e. the kelp will be practically non existent inside the chamber.

It is not possible at this time to predict the extent of the fouling and sedimentation. However its removal would be incorporated into the periodic maintenance schedule.

An important feature of the proposed structure is that the entrance to the water column chamber may be closed off using steel stop logs. These would be located in guides on the front wall of the structure. It would then be possible to dewater the chamber thereby giving 'dry' access for cleaning and inspection.

Research into the ecosystem of the area is presently being carried out by the Dunstaffnage Marine Research Laboratory on the instruction of RPT (see Reference 2). However, this work has initially concentrated on identifying possible problems specific to the HRS device. In view of the different mode of operation of the NEL device this research could now usefully be re-examined.

8. WAVE PARAMETERS

This report makes a preliminary investigation into the stability of the structure and the pressures under the foundation. Accordingly it is considered that the worst cases will be satisfactorily given by adopting an equivalent maximum static design wave approach. In accordance with the Department of Environment Guidance Notes (Reference 4) this would normally be taken as the wave having an average recurrence period of 50 years.

However, at the selected site the maximum design wave parameters are considerably modified by the proximity of the sea bed. The most important effects are shoaling, breaking and refraction. The breaking effect controls the maximum wave height because when the depth decreases to the same order of magnitude as the wave height, waves that are within a certain range of steepness (i.e. height/length) become unstable and break. A CIRIA report (Reference 5) gives the following values for a mean water depth of 16 - 20m.

Wave period	Maximum Wave Height
5 - 10 secs	6 - 14 m
above 10 secs	14 m

9. WAVE LOADINGS

In order to arrive at a method for calculating horizontal wave loadings, it has been assumed that the OWC device acts like a solid vertical breakwater.
This assumption will give conservative results because the water column mechanism will either absorb or attenuate some of the wave energy thereby reducing its peak effect.

Horizontal wave forces can arise from two types of wave. The first is the oscillatory wave which becomes reflected and thereby sets up a standing wave known as a 'clapotis'. The height of the clapotis is generally greater than the height of the approaching wave resulting in maximum overall forces on the structure. The second type of wave is the translatory (i.e. breaking) wave which is more likely to cause critcal forces on local areas of the structure.

Wiegel (Reference 10) states that the theory usually used to predict the pressure in a standing wave is that due to Sainflou (1948). However, he casts doubts on the validity of the formation of the clapotis effect for irregular period waves, but states that measurement of pressure distribution on model and prototype breakwaters has good correlation with the theoretical prediction.

Therefore with the adoption of the equivalent static design wave approach and the undefined effect of the water column mechanism, a simplified pressure diagram for oscillatory waves (see figure 1 - from chapter 24 of Reference 3) has been used to calculate the value of the horizontal wave loading.

9. <u>WAVE LOADINGS</u> Cont'd

Using the 14m maximum wave height given in section 8 at a mean water depth of 18m, Figure 1 (b) gives a value for the horizontal wave force of 370 tonnes per m length of structure acting at a distance of 13.4 m above the sea bed.

10 STABILITY CALCULATIONS

Preliminary calculations for stability against sliding show that the structure cannot mobilise sufficient frictional resistance under the base without the extra downwards force provided by rock anchors. With sufficient anchors installed to provide adequate horizontal resistance the overturning stability is then very high.

It was determined from CP2, Earth Retaining Structures (Reference 11) that the angle of friction under a concrete foundation which has not been cast in-situ may be taken as equal to the angle of wall friction. In this case, assuming a cohesionless soil, the angle of wall friction may be taken as 20°. This value is not reduced by submergence.

Partial safety factors were applied to loads and soil strength parameters in accordance with the FIP Recommendations for the Design and Construction of Concrete Sea Structures (Reference 12). Ultimate values were taken for the factors because it was considered that the wave loading derived in Section 8 represent an ultimate limit state.

The results of the calculations for sliding stability are presented in Table 2. The overall factor of safety against sliding, calculated without incorporating partial safety factors, was found to be 1.53.

10. STABILITY CALCULATIONS (Cont'd)

With the provision of rock anchors as shown on Drawing No. 568/PGA/21/1A the overall factor of safety against overturning was found to be 2.5. Foundation pressure under any part of the structure was found to be less than 0.4 MN/m^2 under the 12 hour storm wave loading. This is very low compared with the rock strength of 10 MN/m^2.

11. COSTS

Output figures for the bottom standing device have been presented in Table 3, along with equivalent figures for the floating OWC, the HRS bottom standing and an optimistic best device (the latter values being derived from reference 14). These figures show that the most likely output from the fixed device may approach the value for the optimistic best device. The main reasons for this improvement are:-

1. Although the total energy of the waves at the inshore site of the fixed device is estimated as 15% less than further offshore it is expected that the greater primary efficiency achieved by fixing the device gives an approximately 170% improvement in output.

2. It is considered that the efficiency of the power chain is considerably improved due to proximity to the shore and direct coupling of the turbine and alternator.

A first order economic evaluation is given in Table 4. This shows that the energy costs from the proposed fixed device may be five times less than the floating device. This is mainly due to:-

1. Lower structure, installation and power take off costs.

2. The considerable reduction in foundation and rock anchor costs compared with the cost of moorings.

3. The lower capital and annual maintenance costs due to the structures being closer to the shore and not requiring mooring replacement.

It has been estimated in Table 4 that the length of device necessary to give an installed capacity of 2 GW is about 32 km. In their preliminary investigation of possible bottom standing sites off Barra, North Uist and South Uist in the Hebrides (reference 2), Rendel Palmer and Tritton concluded that there were between 41 and 48 km of suitable sites for devices orientated towards the prevailing wave fronts. Thus it may be possible to generate up to 3 GW from bottom standing OWC's in that area alone.

12. SUGGESTIONS FOR FURTHER STUDY

Although the preliminary costing given in Section 11 shows that the bottom standing device has prospects of producing power at an acceptable cost, there are a number of areas which require further research to evaluate fully the present proposal.

These are as follows:-

1. The underwater geology of the Hebridean coastline requires detailed investigation of the gneiss bedrock particularly with respect to its extent and soundness.

2. The foundation configuration requires optimisation to reduce the number of rock anchors required and increase the initial installation stability. Study is necessary, also, to determine the constraints on the length and width of the foundations, and their sensitivity to uneveness.

3. To enable the device to be situated at locations other than the Hebridean site assumed in this report, it is necessary to determine what modifications are required to the foundations for sandy and clayey bottoms.

4. Also affecting the detailing of the foundations is the environmental loadings. Much further work is required to confirm the magnitude of the horizontal wave loading, and also to determine the extent of the greater but more localised breaking wave forces.

   The same study should lead to better estimates of the power available at the inshore site. At the time of writing the best estimates vary between a 15% and 40% loss compared to the deeper floating OWC site.

5. The availability of better environmental data will allow the optimisation of the water column size and shape. At the same time the susceptibility of the device to wave slam and water ingress inside the column must be investigated.

6. The effect of seaweed and siltation on the performance of the device must also be investigated.

7. A more detailed costing should be done using the results of the above studies. At the same time updated costs for areas outwith the scope of this report i.e. M & E plant, control systems and transmission costs should also be incorporated.

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TABLE 1

Typical physical and mechanical properties of gneiss.

Porosity	1%
Dry bulk density	26 to 28 kN/m^3
Uniaxial compressive strength	200 MN/m^2
Uniaxial tensile strength	30 MN/m^2
Young's modulus (laboratory value)	50×10^3 MN/m^2
Poissons ratio (laboratory value)	0.1
Presumed bearing value (from CP 2004)	10 MN/m^2

TABLE 2

Values of Lateral Forces

		kN/m
Horizontal wave force		3618
Total structure weight	6158	
less displacement	2440	
Net vertical force	3718	
Frictional resistance mobilised (angle of friction = 20°)		1353
Extra resistance to be provided by anchors		2265

No. of inclined anchors = 56

No. of vertical anchors = 130

Horizontal force provided by inclined anchors		1264
Vertical force due to inclined anchors	2188	
due to vertical anchors	5866	
	8054	
Frictional resistance that can be mobilised (angle of friction = 20°)		2932
Extra resistance provided by anchors		4196

$$\text{o/a factor of safety} = \frac{1353 + 1264 + 2932}{3618} = 1.53$$

TABLE 3

Output Prediction for Bottom Standing Device

Figures for Devices 1 - 3 from RPT presentation notes
for Heathrow Wavepower Workshop, November, 1978.
Figures for Device 4 prepared with assistance from RPT.

No.	KEY: (HIGH ESTIMATE) MOST LIKELY (LOW ESTIMATE)	Annual Apparent Power @ S Uist Buoy	Shallow Water Correction (As captured)	Site Correction (Energy loss & Shielding)	Directionality Correction	Device Capture Efficiency		Power Chain		Power Delivered to Perth
						Based on PM Spectra	Digital Spectrum Correction	Efficiency	Reliability	
	DEVICE	kW/m	fsw	f site	fd	η_d	f digital	η_p	fr	kW/m
1	NEL 78 Reference Design (Floating with Hydraulics)	(46) 42.3 (39)	1.13	(1.15) 1.1 (1.0)	(0.75) 0.65 (0.50)	(0.44) 0.39 (0.34)	0.92	(0.55) 0.37 (0.33)	(0.92) 0.87 (0.80)	(5.7) 4.2 (3.1)
2	HRS	(46) 42.3 (39)	1.13	(1.0) 0.9 (0.7)	(0.75) 0.65 (0.50)	(0.45) 0.33 (0.2)	0.92	(0.60) 0.41 (0.35)	(0.95) 0.92 (0.83)	(4.9) 3.2 (1.9)
3	OPTIMISTIC BEST DEVICE Scenario 2	42.3	1.13	1.1	0.7	0.6	0.92	0.7	0.95	13.5
4	NEL Bottom Standing (No Hydraulics)	(46) 42.3 (39)	1.13	NELx0.85 (0.98) 0.94 (0.85)	(0.75) 0.65 (0.50)	NELx2.0 (0.88) 0.78 (0.68)	0.92	(0.83) 0.65* (0.61)	HRS (0.95) 0.92 (0.83)	(16.0) 12.5 (9.7)

*This figure represents the expected efficiency from a fully developed scheme for power take off.

TABLE 4.1

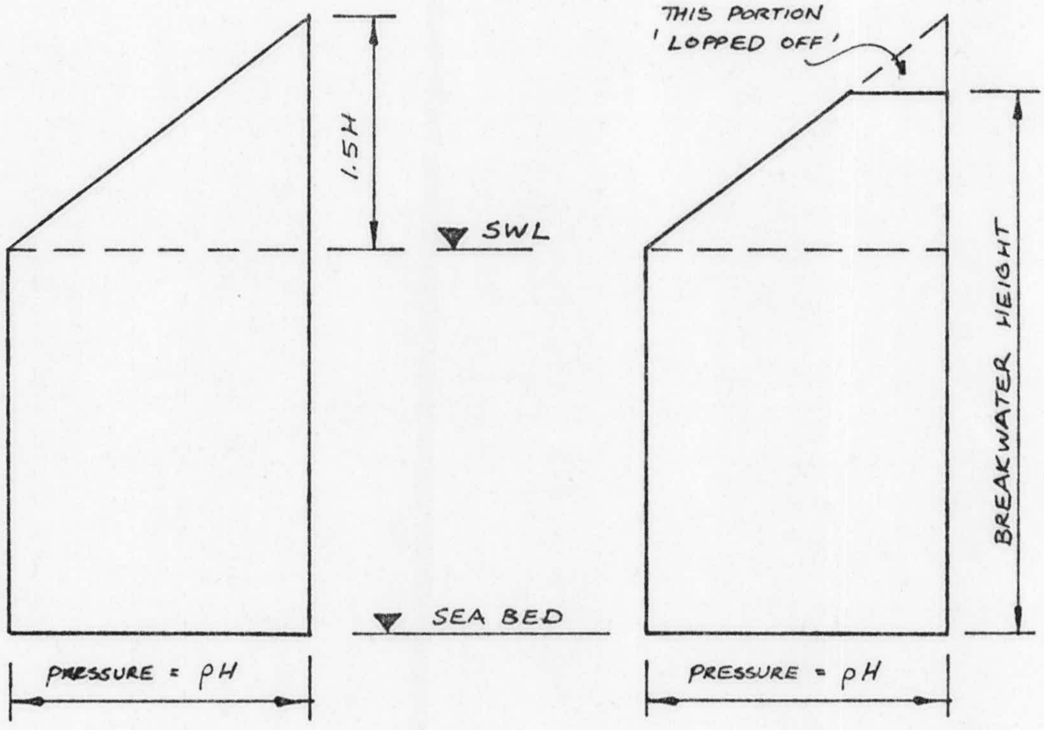
Estimated Costs of Bottom Standing Device

	Bottom Standing		Floating
1. **Installed capacity**			
Average annual output at device	19.9 kW/m		11.7 kW/m
Installed capacity	63.0 kW/m		36.9 kW/m
Unit length	80m		116m
Installed capacity per unit	5MW		4.3MW
2. **Capital cost of one structural unit**			
		£ per m	
Body of Structure	62500		86200
M & E Plant	25000		25800
Tow and install	7500		5200
Foundations and rock anchors	12500	moorings	34500
Power take off	5000		20700
Contingencies	12500		17200
	£125000/m		£189600/m
3. **Capital cost of 2GW power station**			
Installed capacity	63.0 kW/m		36.9kW/m
Length of device	31.7 km		54.2km
No. of structural units	397		467
Capital cost of structures	£3963 x 10^6		£10276 x 10^6
4. **Annual power delivered**			
Average annual output	19.9 kW/m		11.7 kW/m
Power chain efficiency	0.63		0.36
Average annual power delivered	12.5 kW/m		4.2 kW/m
Average annual power (based on 24 x 365 = 8760hrs)	109500 kWh/m		36790 kWh/m
Length of device	31.7 km		54.2 km
Total output per 2GW power station	3471 x 10^6 kWh		1994 x 10^6 kW

TABLE 4.2

	Bottom Standing	Floating
5. Maintenance costs		
Capital cost of maintenance base	£20 x 10^6	£100 x 10^6
Annual maintenance per unit	£1 x 10^5	£5 x 10^5
Annual maintenance per power station	£40 x 10^6	£234 x 10^6
6. Annual costs		
Total capital cost	£3983 x 10^6	£10376 x 10^6
(i) Repayment period 50yrs at 10% compound interest = approx 11% simple interest		
Annual repayment	£438 x 10^6	£1141 x 10^6
Annual maintenance	40 x 10^6	234 x 10^6
	£478 x 10^6	£1375 x 10^6
Cost per unit	13.8p	70.0p
(ii) Repayment period 25yrs at 5% compound interest = approx 7.2% simple interest		
Annual repayment	£287 x 10^6	£747 x 10^6
Annual Maintenance	40 x 10^6	234 x 10^6
	£327 x 10^6	£981 x 10^6
Cost per unit	9.4p	49.2p

Note The figure used for the power chain efficiency is that expected from a fully developed scheme for the power take off component.



(a) BREAKWATER OF UNLIMITED HEIGHT

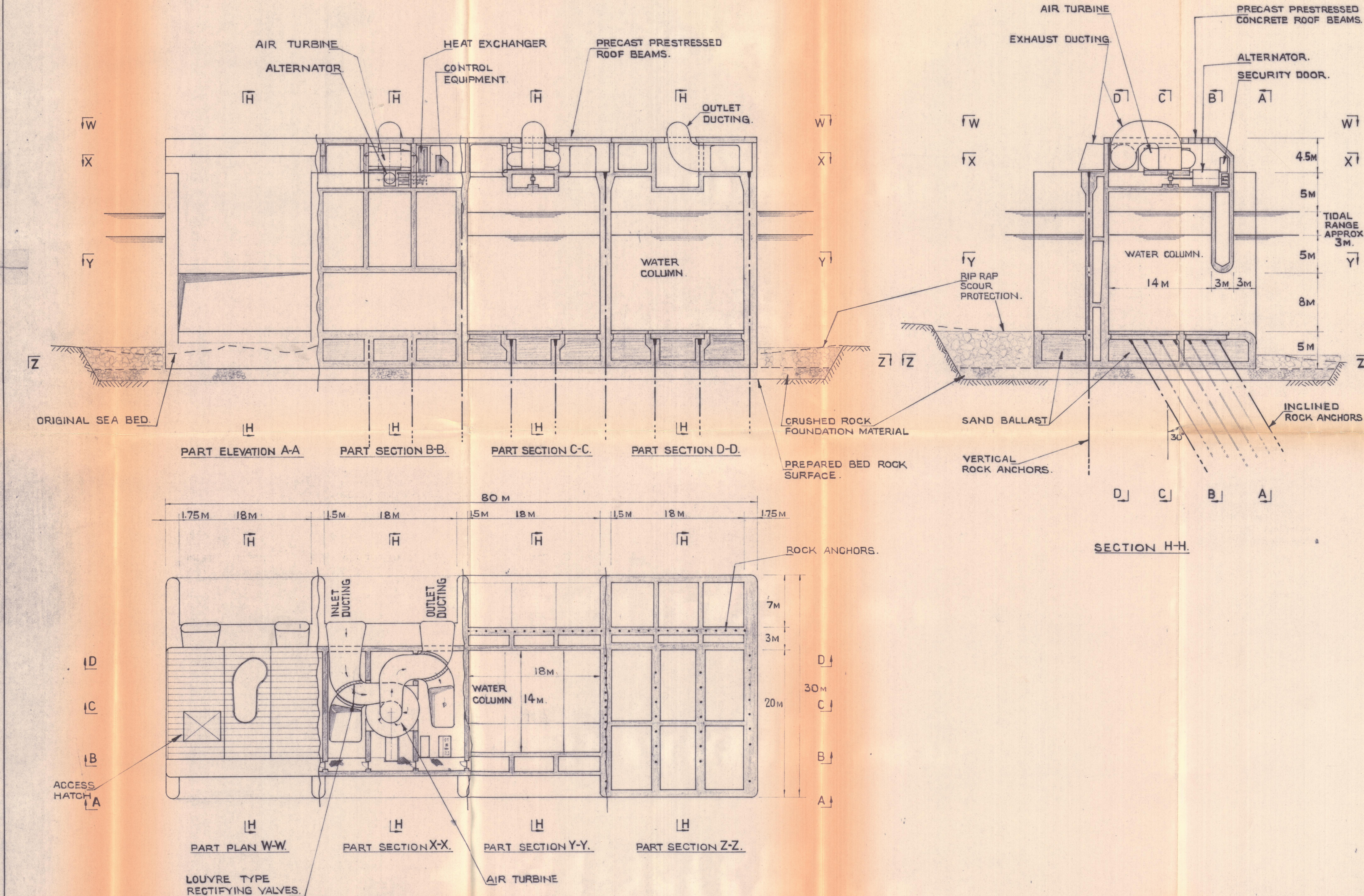
(b) BREAKWATER OF LIMITED HEIGHT

H = MEAN WAVE HEIGHT

ρ = DENSITY OF SEAWATER

FIG 1

PRESSURE DISTRIBUTION FOR OSCILLATORY WAVES



MK.	REVISION	By	Date
A	ROCK ANCHORS ADDED SOME STRUCTURAL CHANGES REDRAWN		SEP'78

NATIONAL ENGINEERING LABORATORY

OSCILLATING WATER COLUMN

PGA 21 BOTTOM STANDING DEVICE.

ROXBURGH & PARTNERS
CONSULTING ENGINEERS
MIRREN HOUSE MAXWELL ST.
PAISLEY SCOTLAND.
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ABERDEEN LONDON INVERNESS

Job No.	Drawing No.	Revision
568	PGA. 21/1	A

Scale 1.250 Date AUG. 78.

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RP Rowan	[illegible]	REPORT PR7: YS DEY2 JUNE 1979

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